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## Finite Element Analysis for Hysteretic Behavior of Thin-Walled CFT Columns with Large Cross Sections

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### Abstract

Thin-walled CFT columns with large cross sections are often used for elevated highway bridge piers in Japan because of their excellent earthquake resistance. This excellent earthquake resistance is primarily due to the restraint of the cyclic local buckling of the thin-walled steel tube caused by the complicated interface action between thin-walled steel tube with diaphragms and in-filled concrete. To predict the performance of thin-walled CFT columns by FEM in a direct manner, it is necessary to model the above interface action as well as the nonlinear behavior of steel tube and confined in-filled concrete properly. Herein, first, an accurate nonlinear FE model is presented to analyze the bi-directional hysteretic behavior of CFT columns. In this model, steel tube and in-filled concrete are represented by geometrically and materially nonlinear shell elements and solids elements, respectively. For the interface between steel and concrete, contact and friction behaviors are considered. Then, a bi-directional cyclic loading experiment is conducted on thin-walled CFT columns to investigate their ultimate behavior. Based on the failure pattern of the in-filled concrete observed in the experiment, a new concept of crush belt is introduced in the modeling of in-filled concrete. The validity of the proposed model is confirmed by the bi-directional cyclic loading experiment.

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Keywords: CFT column; Local buckling; Cyclic loading; FE analysis; Experiment.

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### 1. INTRODUCTION

Concrete-filled thin-walled steel tubular columns referred to hereinafter as thin-walled CFT columns with large cross sections are often used as bridge piers in Japan due to some structural advantages for

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earthquake resistance, i.e., improved strength and ductility together with large energy dissipation capacity, compared with thin-walled hollow steel columns. The strength and ductility of thin-walled steel columns are considerably improved by filling concrete into hollow spaces surrounded by steel tube and diaphragms (**Fig.2**). This excellent performance of thin-walled CFT columns is primarily due to the restraint of the cyclic local buckling of thin-walled steel tube caused by the complicated interface action between thin-walled steel tube with diaphragms and in-filled concrete. Therefore, up to the present, no sufficient numerical methods have been presented to compute the hysteretic behavior of CFT columns in a direct manner by considering the above interaction precisely. In recent decades, the uni-directional cyclic loading experiments have been carried out on thin-walled CFT columns. Following these experimental studies, some FE models have been presented so far to compute their hysteretic behavior. However, these models are mostly based on beam theory (Susantha et al. 2002, Varma et al. 2002) and cannot directly consider the local buckling of steel tube along with the interface action between steel tube and in-filled concrete. In view of the limitations of the beam models mentioned above, the authors recently presented a 3-D FE model for CFT columns (Goto et al. 2009&2010) with considering the local buckling of steel tube, the nonlinear behavior of confined concrete and the proper steel-concrete interaction. This FE model can accurately compute the ultimate behavior of CFT columns under uni-directional cyclic load. Herein, it is examined whether or not the proposed FE model for CFT columns is also applicable to the analysis under bi-directional horizontal cyclic load. For this purpose, a bi-directional cyclic loading experiment has been carried out on thin-walled CFT columns under constant vertical compressive load. Based on the failure pattern of the in-filled concrete observed in this experiment, a modification is made to the original modeling of in-filled concrete. The validity of the modified new model is confirmed by the bi-directional cyclic loading experiment in comparison with the original model.

## 2. MODELING OF THIN-WALLED CFT COLUMNS

### 2.1 Modeling of steel tube

Hysteretic behavior of CFT columns is strongly influenced by the cyclic local buckling behavior of thin-walled steel tube. Therefore, it is necessary to use a geometrically and materially nonlinear shell element to model the steel tube. Herein, the modified 3-surface cyclic plasticity model developed by Goto et al. (2006) is adopted as a constitutive model. This model includes material parameters, such as Young's modulus  $E_s$ , Poisson's ratio  $\nu_s$ , yield stress  $\sigma_y$ , ultimate tensile strength  $\sigma_u$ , original length of yield plateau  $\varepsilon_{yp}$ , plastic modulus  $H_p$  obtained by tensile coupon test, minimum radius of elastic range  $f_b$ , elastic range reduction rate  $\beta$ , elastic range expansion coefficient  $\rho$ , discontinuous coefficient  $\kappa$  and curve-fitting parameter  $\zeta$  for tensile coupon test. The modified 3-surface model is implemented in ABAQUS ver.6.6 by user subroutine feature. As a shell element, 4-node thick shell element (S4R) is used.

### 2.2 Modeling of in-filled concrete

In the conventional concrete models, their multi-axial behavior under compressive stresses is represented by plasticity model, while the behavior under tensile and small compressive stresses is expressed by smeared cracking model. This model, however, often encounters numerical difficulty under cyclic load, when applied to FE model. To circumvent this situation, the concrete damaged plasticity model (Lee and Fenves 1998) implemented in ABAQUS is used herein. However, compared with the conventional smeared cracking model, the concrete damaged plasticity model is considerably approximate in terms of its tensile behavior because an isotropic plasticity is assumed in the tension side. Therefore, this model fails to accurately express the crack opening and crack closing behavior of the in-filled concrete under cyclic load. To make up for this problem, a discrete crack model was inserted at the location of the major horizontal tensile crack that is identified by preliminary analysis (Goto et al.

2009&2010). As a discrete crack model, the so-called contact pair model (ABAQUS 2006) with friction explained in 2.3 was used between the concrete crack surfaces.

Under bi-directional cyclic load, the failure mode of the in-filled concrete is herein investigated by removing the outer steel tube of CFT column specimen after the termination of the bi-directional loading experiment that is explained later in 3. As can be seen from Fig. 1(a), a horizontal crush belt, resulting from the compression failure of the edges of the in-filled concrete cross section, exists surrounding the in-filled concrete cross section at the location of the major horizontal tensile crack. In addition, multiple horizontal cracks are observed in the in-filled concrete. The crush belt is formed by the repetition of the opening and closing of the major crack. Taking into account all the above observed facts regarding the failure mode of the in-filled concrete, the original in-filled concrete model under uni-directional cyclic load (Goto et al. 2009&2010) is modified here to include the crush belt and the multiple discrete cracks, as shown in Fig. 2. The locations of the discrete cracks are determined by preliminary analyses on the corresponding CFT column specimen under the load history to be used in the analysis. The cracks are inserted in an acentric order at a location where axial stress exceeds the tensile strength of concrete until the hysteretic behavior of CFT column model exhibits convergence. The location of the crush belt coincides with that of the primary crack determined by the preliminary analysis. The crush belt at the primary crack is formed in the FE model by partially ignoring the contact action in a certain range from the edge of the in-filled concrete cross section. The width of the crush belt where the contact action is ignored is determined based on the experimental observations. Under the uni-directional cyclic load, however, the crush of concrete is limited only to the upper and lower portions of the edges of the cross section as observed in the failure mode (Fig. 1(b)) and will not spread surrounding the cross section. Therefore, in this case, the new modified FE model with the crush belt proposed for the bi-directional cyclic load may underestimate the strength and ductility of CFT columns under uni-directional cyclic load. The validity of the new modified FE model will be also examined by comparing with the experimental results under uni-directional cyclic load.

### 2.3 Interface modeling between steel and in-filled concrete

There are two types of interfaces in the new FE model. One is the interface between outer surface of in-filled concrete and inner surface of steel tube. The other is the discrete crack surfaces inserted in the modeling of the in-filled concrete. When two surfaces come into contact at the interface, contact pressure as well as friction stress act on their respective surfaces. The contact pressure is calculated by using hard contact model implemented in ABAQUS, while the friction behavior is expressed by Coulomb friction model. In this model, the contact surfaces can carry the resultant shear stress  $\tau_z$  up to the critical shear stress  $\tau_{cr}$  before they start to slip. The critical shear stress  $\tau_{cr}$  proportional to contact pressure  $p$  is expressed as  $\tau_{cr} = \mu p$ , where  $\mu$  is friction co-efficient. When slip occurs,  $\tau_{cr}$  acts on their interface. Herein,  $\mu = 0.2$  is used for steel to concrete interface, while  $\mu = 1.0$  is used for concrete to concrete interface (Goto et al. 2009&2010).

## 3. BI-DIRECTIONAL CYCLIC LOADING EXPERIMENT

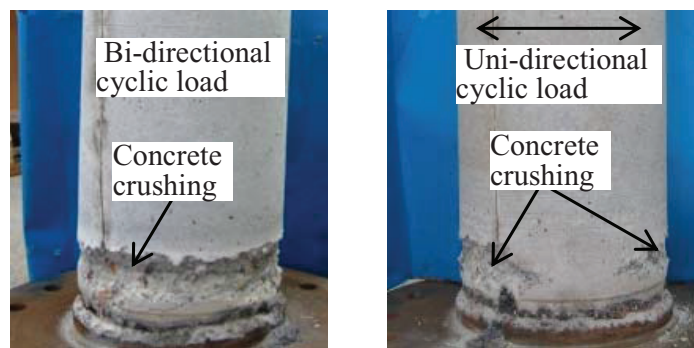
A fully computerized 3-D structural testing system (Fig.3) developed by Obata and Goto (2007) is used to carry out bi-directional cyclic loading experiment on thin-walled CFT columns. The testing system consists of 3 hydraulic actuators and a 3-D hinge (ball joint) to apply arbitrary 3-D load to the top of the column specimen. As a cyclic bi-directional loading program, the displacement-controlled spiral loading illustrated in Fig.4(a) is adopted herein as one of the severest bi-directional loading programs. The displacement controlled spiral load is applied to the top of the column with keeping the vertical compressive load  $P$  constant. The radius  $r$  of the spiral displacement is expressed in terms of the rotational angle  $\theta$  measured from the positive side of the  $x$  axis as  $r = \delta_0 \theta / 2\pi$ , where  $\delta_0$  is the initial yield displacement calculated by  $\delta_0 = (\sigma_y - P/A_s)2h^2 / (3R_s E_s)$ .  $R_s$ ,  $h$  and  $A_s$  are radius, height and

cross section, respectively, of steel tube. In addition to the spiral loading test, the conventional uni-directional cyclic loading test (Fig.4(b)) is also conducted to examine the applicability of the new modified FE model. This is to confirm the versatility of the new model.

The test specimen of the circular CFT column is a cantilever type with fixed base as illustrated in Fig.2(a). This specimen is made of carbon steel pipe (STK400) with 267.4 mm in diameter ( $2R_s$ ). Two diaphragms with 6 mm in thickness are welded to the inside of the steel tube at the intervals of 534.8 mm and 267.4 mm. The concrete is filled up to the first diaphragm. The lower part of the pipe specimens are machined to reduce the thickness  $t_s$  to 4.0 mm from the originals thickness of 8.5 mm such that the radius-to-thickness ratio parameter  $R_t$  of pipe specimen takes the realistic value around 0.09. The geometric properties of test specimens are summarized in Table 1.

#### 4. FEM ANALYSIS

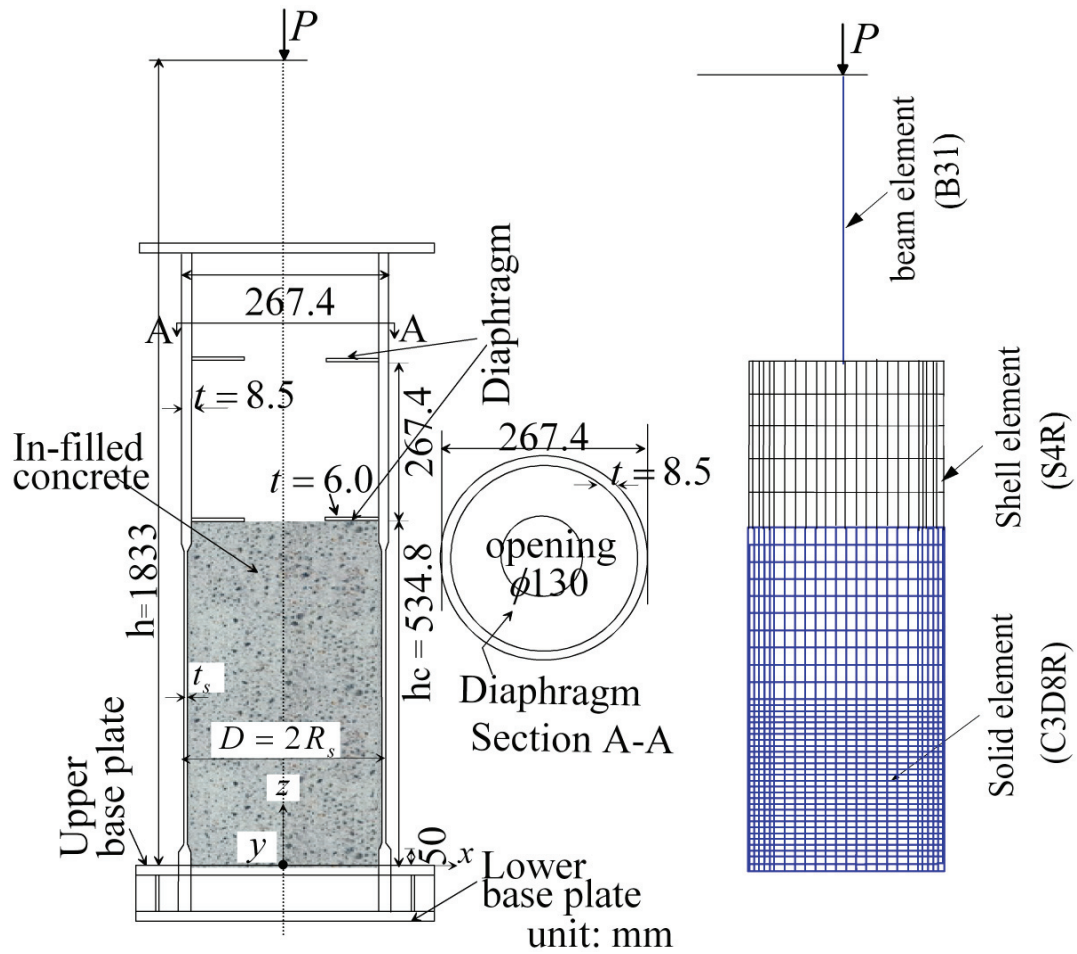
In the FE model for CFT column specimens (**Fig.2(b)**), the lower part of the steel tube with diaphragms is modeled with the 4-node thick shell element (S4R), while the upper part is modeled by the elastic beam element (B31). The concrete core is represented by the 8-node solid element (C3D8R). Four discrete cracks are inserted in the in-filled concrete for the modeling of CFT column specimen (**Fig.2(c)**). In the modeling of crush belt, the averaged width of the crush belt ( $x=18.38$ ) observed from the experiment is considered. A geometrically and materially nonlinear numerical analysis is carried out by the general-purpose finite element package program ABAQUS ver. 6.6. The steel material parameters are identified by a tensile coupon test. The obtained uniaxial stress-strain relation for the material steel is illustrated in Fig. 5. The values of the material parameters for the modified 3-surface model (Goto et al. 2006) are summarized in Table 2. Regarding the material parameters of the concrete damaged plasticity model, the standard values recommended by ABAQUS are used for eccentricity parameter  $e$ , equibiaxial stress ratio  $\sigma_{b0}/\sigma_{c0}$  and shape factor for yield surface  $K_c$ . The Young's modulus  $E_c$ , Poisson's ratio  $\nu_c$  and compressive strength  $f_c$  are determined from the cylinder test. The tensile strength  $\sigma_{t0}$  is determined from splitting test on cylinder specimen. The uniaxial stress-strain curve graphically shown in Fig. 6(a) is applied to the finite elements used for the discretization of the in-filled concrete. This stress-strain curve is determined such that the cylinder specimen model discretized by the finite elements with the size similar to that of the in-filled concrete elements best expresses the compression test results. As the linear stress-crack opening relationship in the post peak range, the negative stiffness in Fig. 6(b) is reduced from the experimental value to ensure better convergence in the nonlinear numerical analysis. Dilation angle  $\psi$  is calibrated by the bi-directional test results of CFT column so that the numerical hysteric curves best fit the test results. The values of material parameters used for the concrete damaged plasticity model are summarized in Table 3. The parameter calibration process for concrete damaged plasticity model is explained elsewhere by Goto et al. (2009&2010).



(a) In-filled concrete (No. 2)

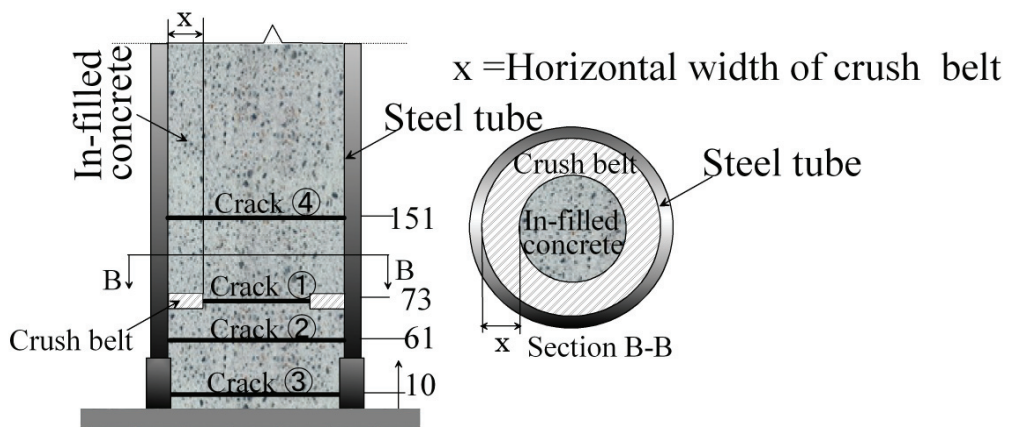
(b) In-filled concrete (No. 1)

Fig. 1: Failure mode of in-filled concrete



(a) CFT column specimen

(b) FE model: Mesh division



(c) FE model: Location of discrete cracks and crush belt in in-filled concrete

Fig. 2: CFT column specimen and FE model

Table 1: Geometric properties of specimens

Specimen	No.1 (CFT)	No.2 (CFT)	No.7 (Hollow)
$h$ (mm)	1833		
$h_c$ (mm)	534.8	534.8	-
$D$ (mm)	262.00	261.90	263.00
$t_s$ (mm)	3.97	3.96	3.99
$R_t$	0.094	0.094	0.093
$\bar{\lambda}$	0.529	0.530	0.528
$P/\sigma_y A_s$	0.15		
$H_0$ (kN)	38.96	38.87	39.60
$\delta_0$ (mm)	9.89	9.88	9.92
Cyclic loading	Uni-directional	Bi-directional (spiral)	

Note:  $\bar{\lambda} = 2h/\pi\sqrt{\sigma_y A_s/E_s I_s}$  (slenderness ratio parameter),  
 $\delta_0 = H_0 h^3/(3E_s I_s)$  (initial yield displacement),  
 $H_0 = (\sigma_y - P/A_s)z_s/h$  (initial yield horizontal force),  
 $R_t = (R_s/t_s)(\sigma_y/E_s)\sqrt{3(1-\nu_s^2)}$  (radius-to-thickness ratio parameter)

Table 2: 3-surface model parameters

Steel	STK400
$E_s$ (GPa)	232.99
$\nu_s$	0.3
$\sigma_y$ (MPa)	399.86
$\sigma_u$ (MPa)	498.16
$\varepsilon_{sp}^0$	0.007
$\beta$	150
$\rho$	2
$\kappa$	2
$\xi$	0.8
$f_b'/\sigma_y$	0.38
$H_n^{mn}$	(*)

(\*)Multilinear curve is used to approximate the hardening behavior(Fig.5)

Table 3: Concrete model parameters

Specimen	No.1	No.2
$E_c$ (GPa)	25.32	27.97
$\nu_c$	0.16	0.17
$f_c'$ (MPa)	30.54	33.68
$\sigma_{t0}$ (MPa)	2.50	2.43
$\sigma_{b0}/\sigma_{c0}$	1.10	
$K_c$	0.70	
$e$	0.20	
$\psi$	10.0°	



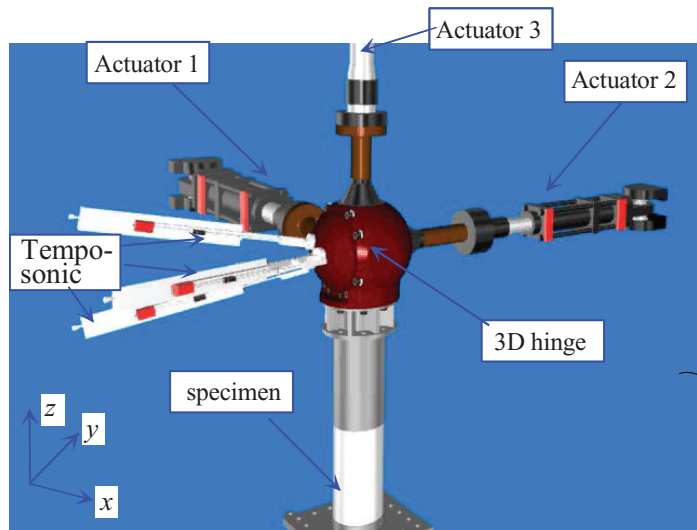


Fig. 3: 3-D structural testing system

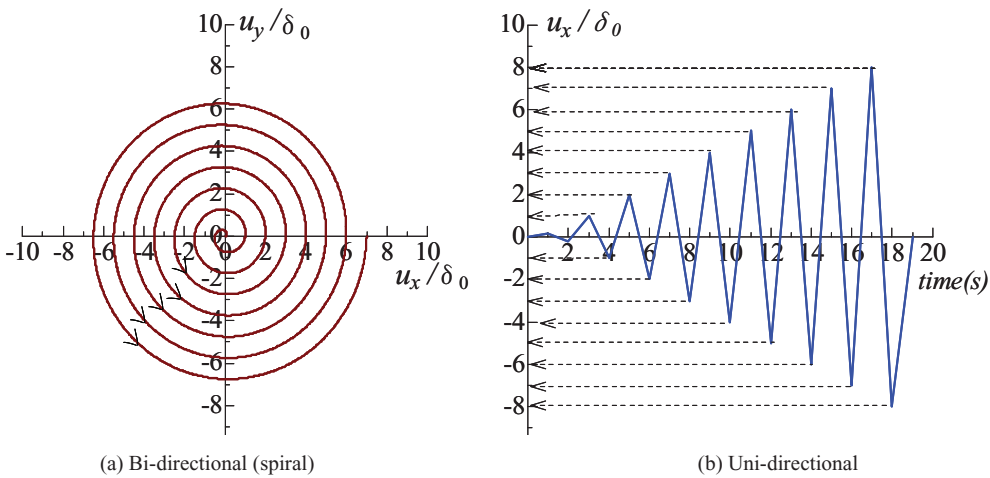


Fig. 4: Cyclic loading program

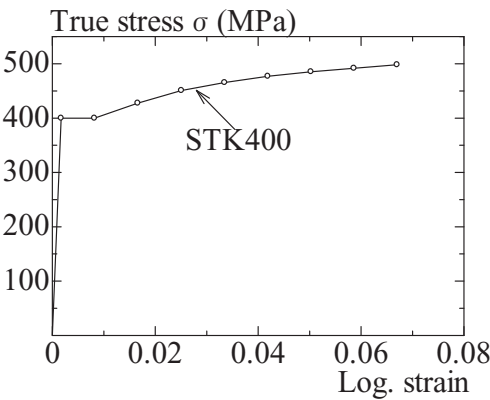


Fig. 5: Uniaxial stress-strain relation of steel

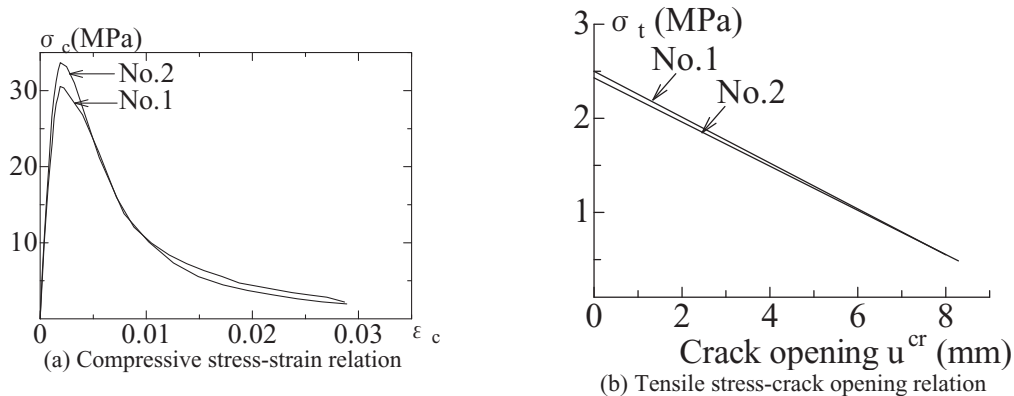


Fig. 6: Uniaxial behavior of concrete

## 5. BI-DIRECTIONAL HYSTERETIC BEHAVIOR OF CFT AND HOLLOW COLUMNS

The bi-directional cyclic loading test results on the CFT column specimen are compared to the numerical results computed by the original FE model without crush belt in **Fig. 7**, in terms of the horizontal restoring force-displacement relations for the directions of the x- and y- axes in addition to the vertical displacement-resultant horizontal displacement relations. For comparison, the numerical results obtained by the new FE model with crush belt are shown in **Fig. 8**. In this new FE model, the multiple cracks are introduced similar to the original FE model. For reference, the bi-directional cyclic loading test results for the hollow column are shown in **Fig. 9**, compared with the numerical results obtained by the nonlinear FEM shell analysis.

It is observed from **Figs. 7** and **8** that both the proposed new model with crush belt and the original model have an acceptable accuracy to predict the bi-directional hysteretic behavior of thin-walled CFT columns. The new model more properly predicts the horizontal hysteretic behavior up to the maximum restoring force and the vertical displacement at the last cycle of spiral load, compared with the original model without crush belt (**Fig.7**). However, the new model slightly overestimates the post peak negative stiffness of the envelopes of the horizontal hysteretic curves. From the comparison between **Figs. 8** and **9**, the strength and ductility of the CFT column under bi-directional cyclic load are considerably increased from those of the hollow column.

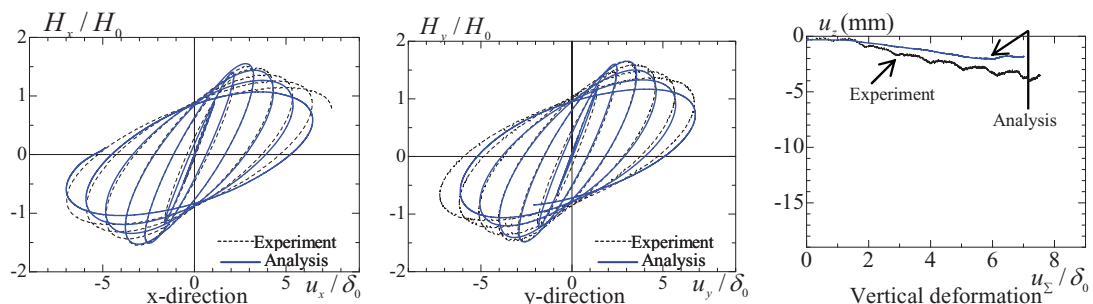


Fig. 7: Bi-directional hysteretic behaviour of CFT columns (No. 2) without crush belt



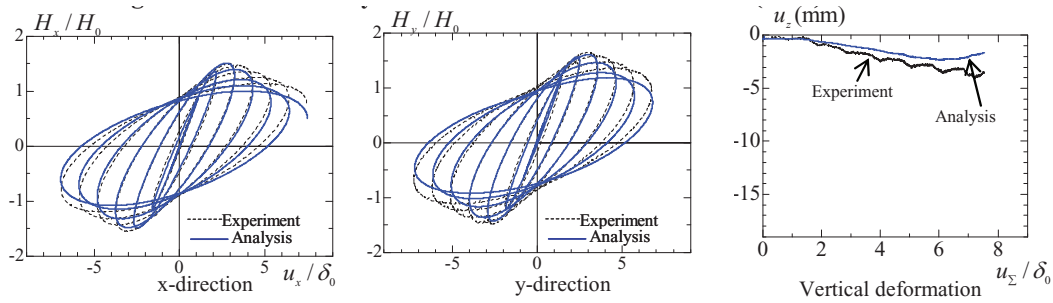


Fig. 8: Bi-directional hysteretic behaviour of CFT columns (No. 2) with crush belt

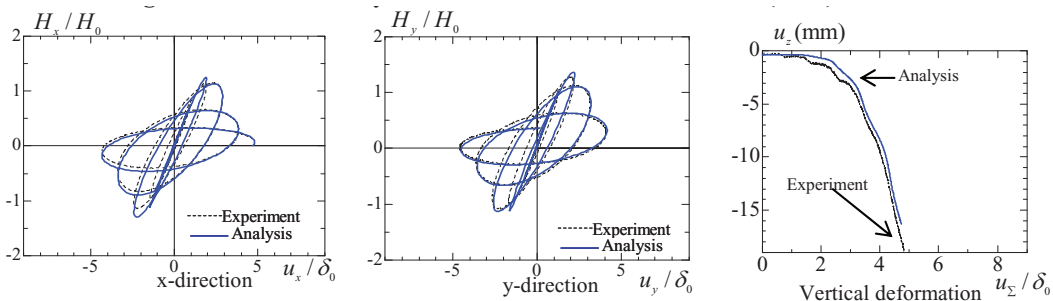


Fig. 9: Bi-directional hysteretic behaviour of hollow columns (No. 7)

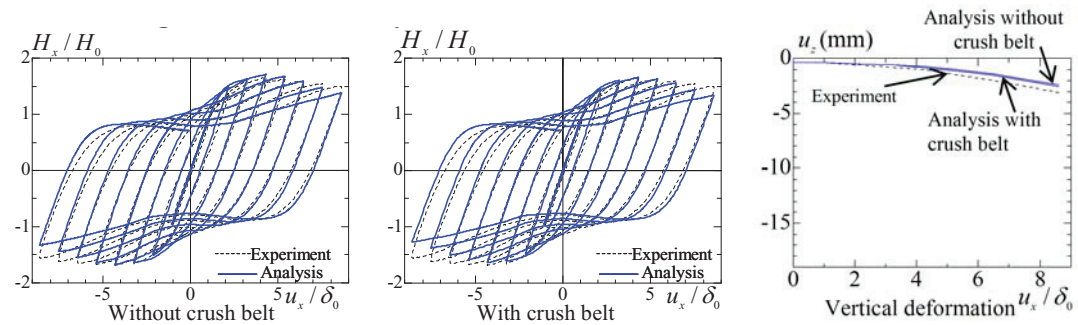


Fig. 10: Uni-directional hysteretic behaviour of CFT columns (No. 1)

Since the crush of in-filled concrete under uni-directional cyclic load is limited to the upper and lower portions of edges of the cross section (Fig.1), the new FE model with the crush belt may underestimate the strength and ductility of CFT columns under uni-directional cyclic load. Therefore, the uni-directional cyclic loading test results on CFT columns are compared in Fig. 10 with the numerical results obtained by the two in-filled concrete models. As can be seen from Fig. 10, the accuracy of the new FE model with the crush belt is similar to that of the original FE model without the crush belt. It can be said from these results that the crush belt has little effect on the hysteretic behavior of the CFT columns under uni-directional cyclic load and may be used as a versatile modeling of in-filled concrete under arbitrary cyclic load.

## 6. SUMMARY AND CONCLUSIONS

In view of practical application to the seismic performance evaluation of CFT columns with large cross sections, an accurate and numerically stable FE model is presented to compute their bi-directional hysteretic behavior in a direct manner. In this model, cyclic local buckling of steel tube, nonlinear

behavior of confined concrete and interface action between steel tube and in-filled concrete are fully considered. Among others, special considerations are made to the modeling of in-filled concrete. The behavior of the in-filled concrete is herein expressed by the concrete damaged plasticity model combined with the discrete crack model and the newly proposed crush belt model. The inclusion of the discrete crack model is indispensable since the tensile behavior of concrete and the increase of volumetric strain cannot be accurately expressed by the concrete damaged plasticity model. Under the bi-directional cyclic load, the crush belt model will be needed in order to express more properly the horizontal behavior up to the strength as well as the vertical displacement, respectively, of CFT columns. However, the new model slightly overestimates the deterioration in the post peak range of the envelopes of the horizontal hysteretic curves. The accuracy of the new model is also confirmed by the test results under uni-directional cyclic load.

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